DESIGN
OF A
POWER DEVELOPMENT DAM

...BY...

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THIS IS TO CERTIFY THAT THE THESIS PREPARED UNDER MY SUPERVISION BY

HENRY CAMILLE DADANT

ENTITLED DESIGN OF A POWER DEVELOPMENT DAM.

IS APPROVED BY ME AS FULFILLING THIS PART OF THE REQUIREMENTS FOR THE DEGREE

OF Bachelor of Science in Civil Engineering.

Ira O Baker

HEAD OF DEPARTMENT OF Civil Engineering

Introduction

Many projects for the developement of the water power of the Des Moines rapids have been advanced for four decades, but none have been carried out. It has recently been proposed to build a dam across the river at the foot of the rapids, and the writer has chosen as the subject for his thesis the design and the estimate of the cost of this dam.

History of the Water Power Project

The Des Moines or Lower rapids of the Mississippi river are situated about 5 miles to the northeast of the mouth of the Des Moines river. The cities of Keokuk, Iowa, and of Hamilton, Illinois, are on the west and east sides of the river respectively, near the foot of these rapids; the rapids are about 12 miles long, the head being situated near Montrose on the Iowa side and Nauvoo on the Illinois side. At the rapids the Mississippi river has cut its way thru the carboniferous or mountain limestone to a depth of about 150 feet and nearly 200 feet below the adjacent prairies. The cut is practically unvarying in width, being 7/8 of a mile between bluffs, while the river bed is quite uniformly 3/4 of a mile wide.

The rapids in their practically uniform descent are broken by several reefs, commonly known as chains, of very hard and tough limestone extending across the river, the principal ones being the Lower, the English, the Lamallees, the Spanish and the Upper chain. The greatest rate of fall occurs at the Lower chain, where it is 1.47 feet in 1,000 feet. The total fall during low water is about 23 feet, and during high water only about 16 feet. Any roughness of the surface of the water or any noise due to the waters rushing over the rapids is noticeable only during low water. During high water the rapids are virtually drowned out and are entirely unnoticeable.

The difficulties of navigating the rapids at ordinary stages and their unnavigability during low water caused early efforts to be made by the government to provide some means by which they could be navigated at all times. It was first contemplated to widen and deepen the existing crooked natural channel. With the intention of carrying out this project, work began in 1837 and continued until 1866. During this time 25,000 cubic yards of stone were excavated at an aggregate cost of $335,000. The great cost and impracticability of this scheme, as well as its inadequacy if completed, then became evident; and in 1867 a board composed of United States engineers met at Keokuk and decided that the improvement should be partially effected by constructing a lateral canal along the Iowa shore from Keokuk to Nashville, a distance of 7.6 miles, and also that the improvement should be completed from Nashville to Montrose by cutting a channel thru the rock. The canal was begun in 1867, was opened for traffic in 1877, and has been almost continously in use.

It is interesting to note that before the adoption of the method of improving the rapids by a canal along the Iowa shore, certain parties in Illinois advocated various other projects of improving the rapids and also of developing the water power. Of these projects the principal ones were: (1) the construction of three dams across the river, (2) the building of a system of wing dams and sluices, and (3) the construction of a canal on the Illinois side. These projects were rejected by the board as being impracticable or inferior to the one adopted, namely, a lateral canal on the Iowa side. While considering the project of a canal on the Illinois side, the board of engineers conceded that the Illinois side offered the better natural advantages for the developement of water power; but held that the primary object of the general government was to facilitate navigation and to do this at the least possible cost. Had modern facilities in engineering construction, in water power developement, and in electrical transmission of power existed when the construction of a canal on the Iowa side was proposed, there is but little doubt that some project of a similar nature to the one to be treated herein would have been chosen in preference to the plan then adopted, inasmuch as not only would a dam at the foot of the rapids have been better and cheaper, but from 50,000 to 60,000 horse power would have been developed.

Various attempts have been made by private parties in later years to promote schemes for developing the water power at the rapids, but practically nothing was achived until the Keokuk and Hamilton Water Power Company was organized in 1900. This company, composed of the most prominent men of the cities of Hamilton and Keokuk, was incorporated under the laws of the state of Illinois to promote the development of the water power. The company first secured a franchise from the government, which gave them the right to devel-op the water power of the Des Moines rapids in any manner they desired, as long as the structures built for this purpose "did not in any way interfere with the existing low water channel over the rapids or with the interests of navigation". A civil engineer, Mr. Lyman E. Cooley of Chicago, was then employed to determine the best method of developing the water power and to report on its feasibility as an engineering and financial undertaking. From the report submitted by Mr. Cooley it became evident to the Water Power Company that the most feasible plan was a dam at the foot of the rapids extending across the river from shore to shore. As the construction of such a dam was prohibited by the limitations of the franchise, steps were taken to get the government to carry out this project, which would mean the replacement of the existing canal with its three locks by a dam at the foot of the rapids with a single lock. Accordingly in 1903 a board of United States engineers convened at Keokuk to determine whether it would be advisable for the United States government to construct the proposed dam. The board found that there were several advantages to be gained for navigation by replacing the canal and the three locks by a dam and one lock, but decided that the advantages were not enough to justify the United States in the expense necessary to carry out the new project. The board stated, however, that the government would be willing to allow private parties to construct a dam at the foot of the rapids, the details for the same to be subject to the approval of the Secretary of War and of the Chief of Engineers.

No further action has been taken by the Water Power Company to carry out the project, nor is it known what steps will be taken next to promote the development of the water power of the Des Moines rapids.

It is here proposed to consider the construction of the single dam, which has been discussed above.

The Dam

Location. The location of the dam is to be at the foot of the rapids and to extend across the river in a line perpendicular to the general course of the stream. The line will lie approximately half way between the lower lock of the canal and the mouth of Cheney creek, — see Plate I. There is no detailed contour map of this portion of the river available from which the most desirable location for the dam can be chosen. The provisions for power installation will probably somewhat affect the choice of the exact location of the dam, but the requirements of the power installation is not a part of the problem here considered, altho some thought has been given to this phase of the general subject in choosing the location and in arranging some of the details of the design.

For the present it will be assumed that the dam has the location shown in Plate I, as a slight change of position will not affect its section or its length materially. The west abutment will be 2,100 feet above the lower lock of the government canal. From thence the structure will extend across the river, east 4,360 feet to a point 2,600 feet S. 14° W. of the limestone bluff just above Cheney creek, and thence to the east abutment located in the limestone bluff above mentioned.

There will be a lock in the extreme west end of the dam which, together with the west abutment, will occupy 600 feet of the length of the dam. A breach in the dam of 1,200 feet, beginning 1,500 feet from the west abutment or 900 feet east of the east wall of the lock, will be left for the purpose of providing a wasteway.

Height of Dam. The approximate height of the dam necessary to bring the crest above the level of the water to be maintained back of the dam, is given below, and was computed by the use of elevations given on Chart No. 136 of the Mississippi River Commission. The average elevation of the river bed at the dam site is about 485 feet, and the dam will be founded 3 feet below this. An elevation of 517 feet back of the dam corresponds to a head of 32 feet. This head will cause no large amount of valuable land to be flooded above the rapids. The financial advantages accruing from a greater head than this would probably not be justifiable on account of an increase in the first cost due to a rapid increase of land damage contemporary with a small increase in head. The dam proper will be 38 feet high or 3 feet above the level of the water back of it and will be carried 3 or 4 feet higher by parapet walls built above each face, sufficient width being left between them for a walkway.

The elevation of the bed of the river at the upper end of the east portion of the dam is greater than at the main portion. The difference in these elevations, however, is small, and the height of that portion of the dam extending from the Illinois bluffs above Cheney creek 2,600 feet S. 14° W. will be 38 feet or the same as the main portion.

Section of Dam. The cross-section of the dam is shown on the following page. After repeated trials this section was selected. It is of the well known pure-gravity triangular type, which consists essentially of a triangle surmounted by a rectangle. The object of the increased width on top due to the latter is to resist the action of ice, waves, wind, etc. The total height of the dam is 38 feet, or 35 feet above the river bed, and the head of water back of it is 32 feet. The width on top is 9 feet, and at the bottom 29.94 feet. The total area of the cross-section is 663.11 square feet.

In designing the foregoing section a length of only one foot was considered. The weight of the material (concrete) was assumed to be 145 pounds per cubic foot, which is probably a little small. The vertical pressure on the upper face of the dam was not considered and therefore the actual stability is greater than the computed.

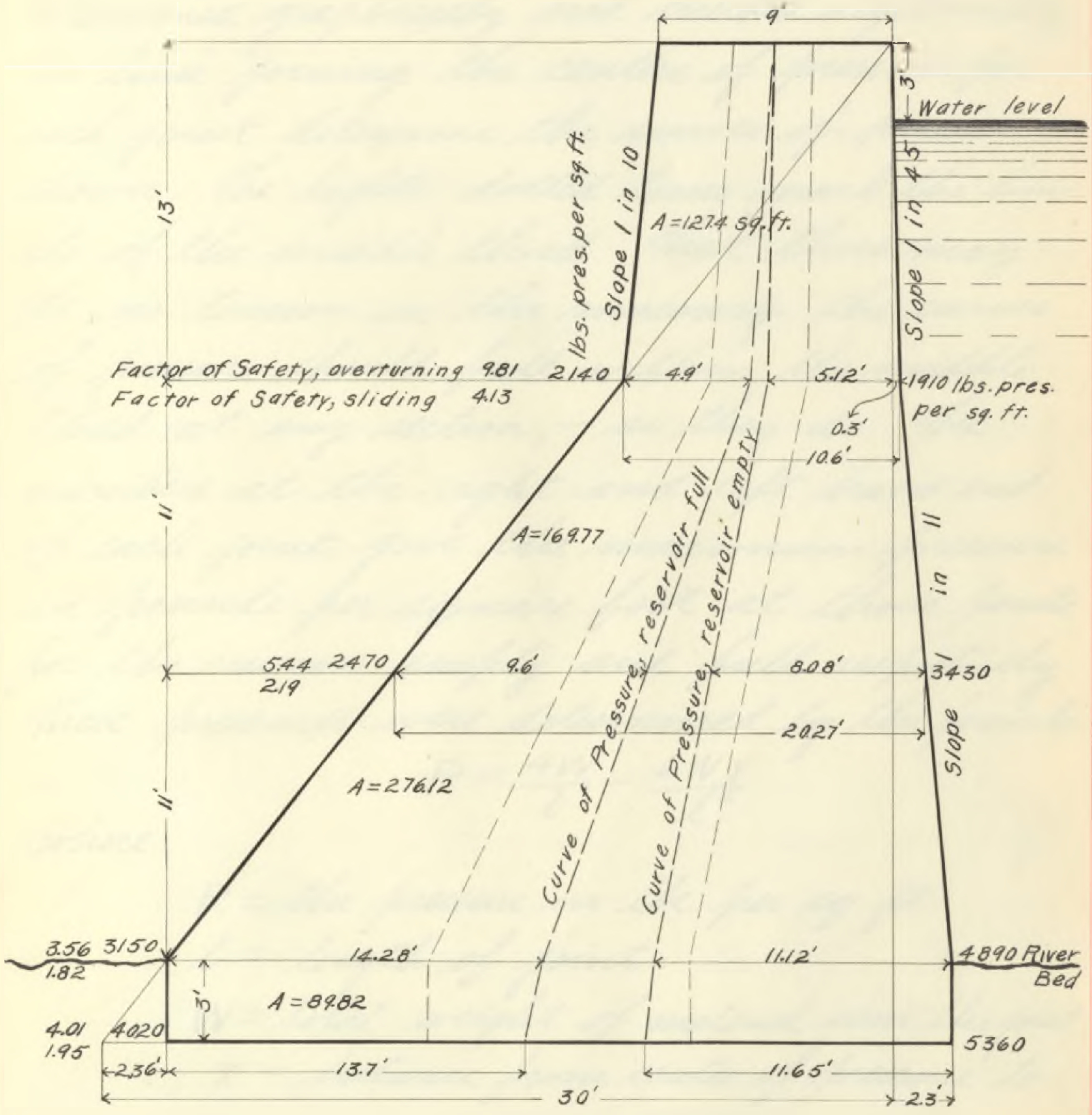


Fig 1. — SECTION OF DAM

Scale 1" = 60'

The centers of gravity and resultant centers of pressure for the several sections were determined graphically and checked algebraically. The lines joining the centers of pressure for each joint determine the curves of pressure shown. The lighter dotted lines mark the limits of the middle third. That there may be no tension in the masonry, the curves of pressure should fall within the middle third at any section, — as they do. The numbers at the right and left hand end of each joint give the maximum pressure in pounds per square foot at those points for the reservoir empty and full respectively. These pressures were determined by the formula:

$$P = \frac{4W}{l} - \frac{6W\bar{x}}{l^2}$$

where

P = the pressure in lbs. per sq. ft.

l = length of joint.

W = total weight of material above the joint.

$\bar{x}$ = distance from center of pressure to edge of joint.

The factors of safety against overturning and against sliding for the different joints are given in Fig 1. The coefficients of friction used was 0.65 at the bottom, and 0.70 for the other sections.

Theoretically the pressures at the right and left hand end of any joint should be equal. As the section is designed the pressures along the back face are greater than those along the front, the former occurring when the reservoir is empty and the latter when the reservoir is full. As the reservoir will always be full except during the time of constructing or repairing the dam, the smaller pressures along the lower face is permissible.

Abutments. An abutment at the extreme west end of the dam will virtually be a part of the lock, and hence will not be considered here, as the design of the lock will not be attempted. An abutment will be needed at the west end of the dam proper, or adjoining the east side of the lock and another at the north end of the east portion of the dam. The former will be 50 feet long, and the latter 100 feet. A polygonal abutment will be built at the junction of the main and the east portion of the dam. The sections and elevations of the abutments are shown on the following page and need no further explanation.

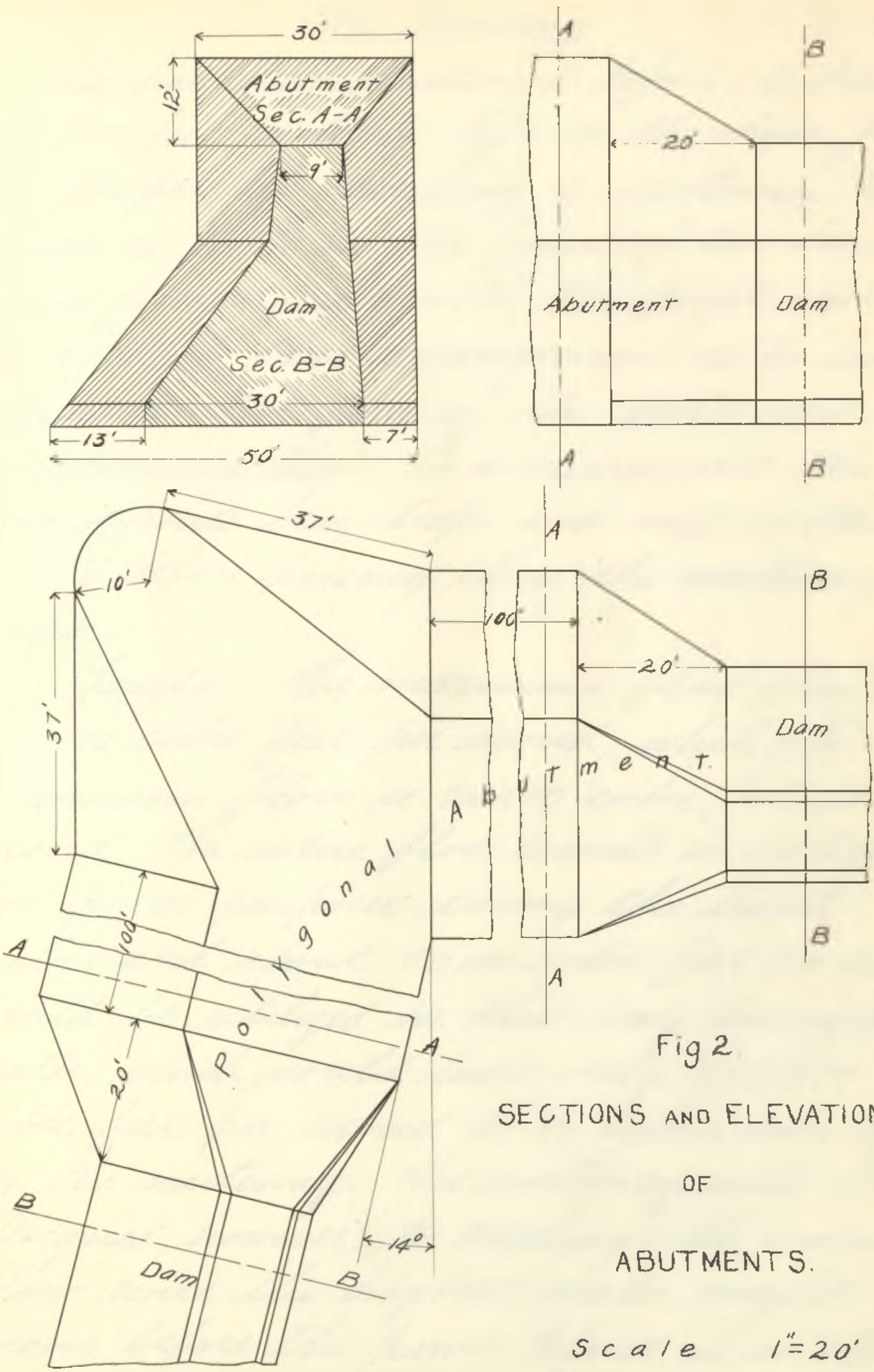


Fig 2

SECTIONS AND ELEVATIONS

OF

ABUTMENTS.

Scale 1" = 20'

The Wasteway

As has been mentioned before, a breach of 1,200 feet is to be left in the dam for the purpose of providing a wasteway. This breach is to be in the form of two bear-trap dams forming a V with the point upstream. The object of this arrangement is to lessen the destructive force of the water and ice, by allowing them to discharge over the two opposite bear-traps and meet within the V before passing thru the wasteway proper.

Length. The maximum flood flow is 372,500 cubic feet per second, while the minimum flow is 20,000 cubic feet per second. The entire flow would be utilized for power purposes during the lowest water, and about 50,000 cubic feet per second would be utilized in this way during high water, which would leave only 322,500 cubic feet per second to be taken care of by the wasteway. The wasteway will be designed, however, to discharge the maximum flow since the highest water might occur before the power plant is in operation or at some time when it is not in use.

Few, if any, experiments for discharge thru large suppressed wasteways are on record, and in selecting the length of wasteway needed the values given by the different formulas should serve merely as guides to the judgment. Two formulas will be considered.

1. The following method of determining the length is based on the results of experiments made by Francis with suppressed weirs. In the formula

$$Q = 3.33\,L\,(nH)^{\frac{3}{2}}$$

Q = the discharge in cu. ft. per sec.

L = length of weir in ft.

n = a coefficient depending upon the ratio of H' to H, in which H' is the difference in elevation between the crest of the weir and the up-stream surface and H is the difference in elevation between the crest of the weir and the down-stream surface.

For the proposed dam

$$H = 32 \text{ ft.},$$

$$H' = 20 \text{ ft.},$$

$$\frac{H'}{H} = \frac{20}{32} = 0.625$$

From a table given by Francis for different values of $\frac{H'}{H}$

$$n = 0.833$$

Allowing an error of three units in the second decimal place, we may assume

$$n = 0.803, \text{ or say } 0.800$$

As stated above

$$Q = 372,500 \text{ cu. ft. per sec.}$$

Then by transposing the above formula and substituting

$$L = \frac{372,500}{3.33(0.8 \times 32)^{\frac{3}{2}}}$$

$$= 865 \text{ ft.}$$

2. The following method of determining the length of wasteway is based on the results of experiments made by Fteley and Stearns with suppressed weirs. In the formula $Q = mL\,(H+\frac{1}{2}H')(H-H')^{\frac{1}{2}}$, $m = c\,\frac{2}{3}\sqrt{2g}$, c being a coefficient of discharge depending upon $\frac{H'}{H}$ and g the accelerating force of gravity. From a table given by Fteley and Stearns for different values of $\frac{H'}{H}$, we find

$$m = 3.09$$

Then substituting this in the formula above and solving

$$L = \frac{372{,}500}{3.09 \times (32+10)(32-20)^{\frac{1}{2}}}$$

$$= 831 \text{ ft.}$$

Table I gives the discharge computed by the two formulae above for different values of L.

Table I

Maximum Discharge for Different Lengths of Wasteway

Length in feet	1 Francis formula	2 Freley & Stearns formula
800	344 500	368 700
831	358 000	372 500
865	372 500	388 000
900	387 700	404 000
1 000	431 200	449 000
1 100	474 500	494 000
1 200	518 000	539 000

From this table it appears that a wasteway 900 feet long would be ample. There are several factors, however, which would affect the computed capacity of the wasteway. Those which would tend to increase its computed capacity are the velocity of approach and a properly curved end of the wasteway; while the only factor tending to decrease the computed value is the effect of the friction of the water on the bottom and sides of the wasteway. The decreasing effect of the latter on the discharging capacity of a spillway is greater than the decreasing effect due to the friction of the water discharging over the crest of a weir. As the coefficients used in the formulas above were based on the results of experiments made on weirs, the actual discharge will tend to be less than the computed. The effect of the friction of the water discharging thru the wasteway, will surely more than counterbalance the factors tending to increase the computed discharge and hence a wasteway more than 900 feet long will be needed.

To obtain a discharge of 372,500 cubic feet per second with a wasteway 900 feet long, it would be necessary to lower the bear-traps to the river bed. The advantages of the bear-traps would then not be realized, but the maximum flood flow does not occur simultaneously with the passing of the ice and consequently the bear-traps are not needed, and lowering them does no harm. However, the necessity of repairing a section of the bear-trap dams might occur during the time of the highest water which might affect the discharging capacity of the wasteway materially.

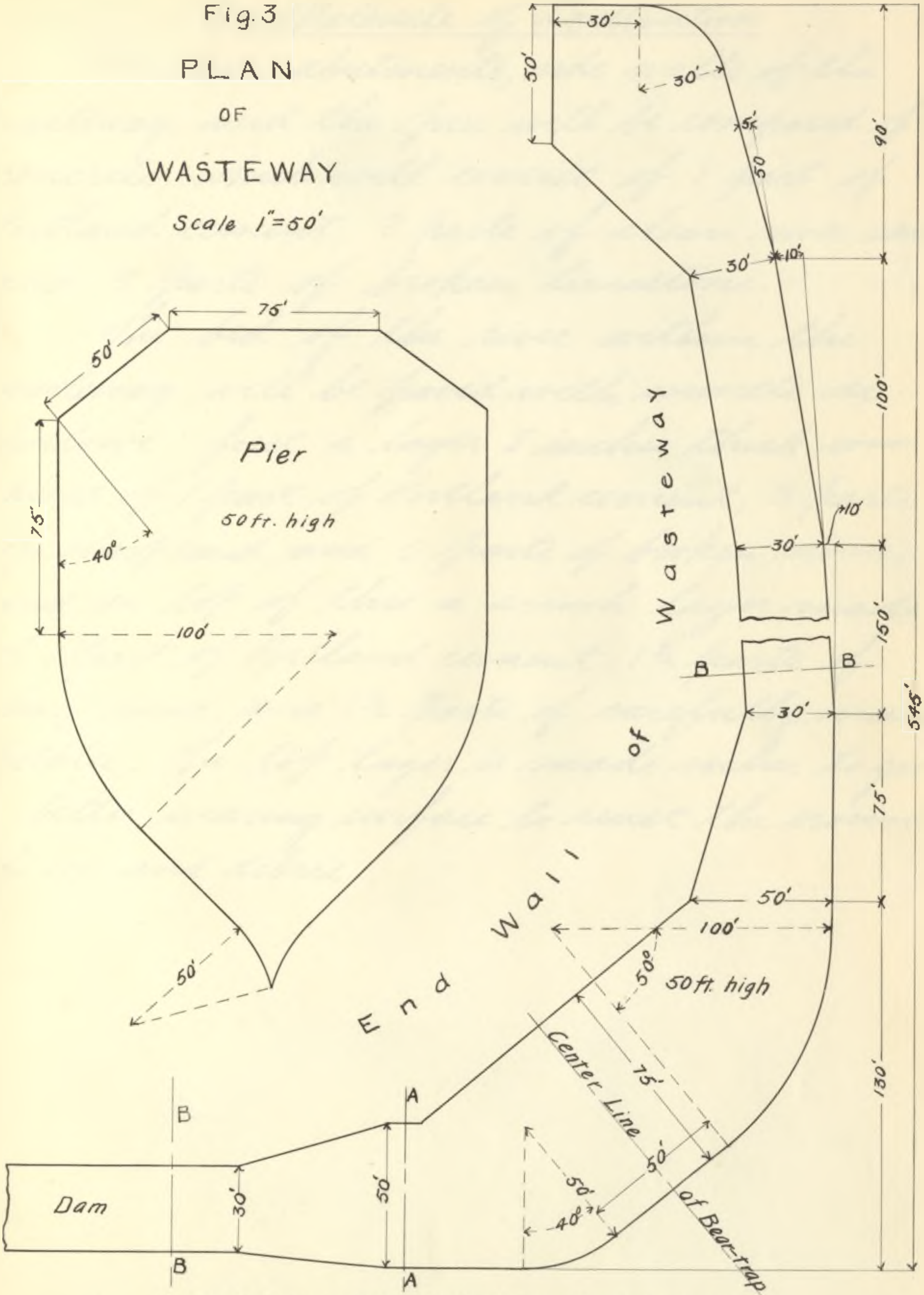
A length of wasteway of 1,000 feet will give a velocity of nearly 11.6 feet per second; but it is not desirable to have a velocity greater than 10 feet per second thru the spillway, which would require the wasteway to be about 1,160 feet long. After carefully considering the results of the formulas given above and the factors tending to increase and decrease the capacity of the wasteway, it was decided to use a wasteway 1,200 feet long.

Plan of Wasteway. The general plan of the wasteway is shown on Plate I, and a more detailed drawing of the pier at the upper end of the V and of one of the side walls of the wasteway is shown on page 21. The bear trap dams are each 925 feet long. They should be built in five sections and be controlled by undermined power. The design of the bear traps will not be included in this volume.

Fig. 3

PLAN OF WASTEWAY

Scale 1"=50'



Note: Sections A-A and B-B are shown on page 14.
For a general plan of the wasteway see plate I.

The Materials of Construction

The dam, abutments, side walls of the wasteway, and the pier will be composed of concrete, which will consist of 1 part of Portland cement, 3 parts of clean river sand, and 5 parts of broken limestone.

The bed of the river within the wasteway will be paved with concrete as follows: first a layer 8 inches thick composed of 1 part of Portland cement, 3 parts of sharp sand, and 5 parts of broken stone, and on top of this a 4-inch layer made of 1 part of Portland cement, 1½ parts of sharp sand, and 1½ parts of carefully screened pebbles. The top layer is made richer to give a better wearing surface to resist the erosion of ice and debris.

Estimate of Cost.

Dam and Wasteway

286,662	cu. yds	concrete	@ $5.00	$1,433,310.00
12,151	" "	"	" 6.50	78,981.50
25,353	" "	rock excavation	" 1.00	25,353.00
6,850	ft.	coffer-dam	" 9.00	61,650.00
				$1,599,294.50

Bear-trap dams

1850	ft.	bear-trap dams	@ $1810.00	$3,330,000.00
8565	cu. yds	rock excavation	" 1.00	8,565.00
				$3,338,565.00

Total $4,937,859.50

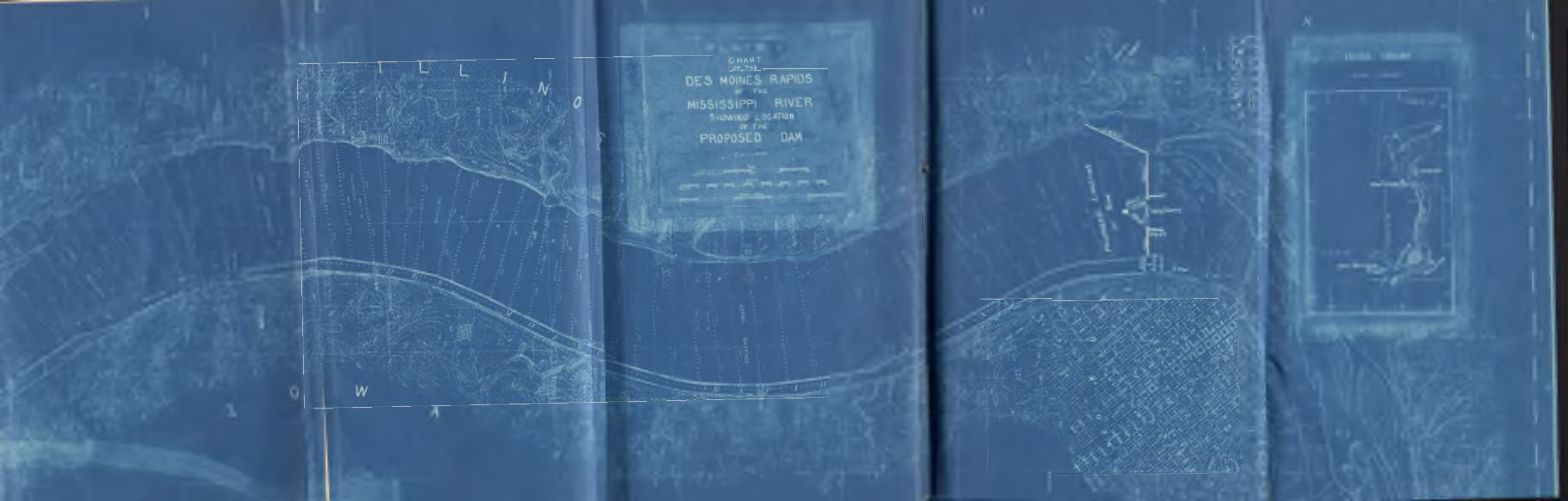


CHART
OF THE
DES MOINES RAPIDS
OF THE
MISSISSIPPI RIVER
SHOWING LOCATION
OF THE
PROPOSED DAM